# LACONIA ▲
### DEVELOPMENT LLC
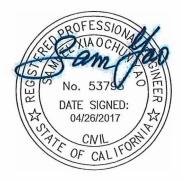
# Lᵒ
# LATITUDE
LUXURY HOMES ON SAN FRANCISCO BAY

CONDITION
ASSESSMENT AND
DESIGN CRITERIA FOR
STRUCTURAL
EVALUATION OF
LATITUDE WHARF

POINT RICHMOND,
CALIFORNIA

24 May 2017 ECRB Meeting

SGH Project 167573

**SIMPSON GUMPERTZ & HEGER** ▶

Engineering of Structures
and Building Enclosures

# 4. SHORELINE PROTECTION ASSESSMENT

## 4.1 BATHYMETRY AND DATUM

Based on National Oceanic and Atmosphere Administration (NOAA) Station 9414863 at Richmond, California, water levels and tidal datum at the site are summarized below in Table 4-1. The datum is referenced from Mean Lower Low Water (MLLW).

### Table 4-1: Tide Conditions

| Tide condition | Elevation (NAVD88) |
|---|---|
| Max. Observed | +8.65 ft. |
| Mean Higher High Water (MHHW) | +6.06 ft. |
| Mean Sea Level (MSL) | +3.26 ft. |
| Mean Lower Low Water (MLLW) | 0.00 ft. |
| Min. Observed | -2.51 ft. |
| Base Flood Elevation (BFE) | +11.00 ft |
| BFE +3' Sea Level Rise | +14.00 ft |

## 4.2 WIND CONDITIONS

Wind data is obtained from the NOAA database for the Naval Air Base Station in Alameda as a source with enough data points to accurately predict possible wind velocities. Local stations surrounding the project site in Richmond are available but did not provide enough data points for determining an extreme value as a statistically returning event. The data from the selected station will be considered representative for the San Francisco Bay, including the project site. Although directional wind data is available for the station, only overall wind speeds have been used for the statistical analysis to avoid errors from possible differing local effects at the site.

### Table 4-2: Extreme Wind Event

| Return Period | 30 Second Wind Speed |
|---|---|
| 100-Year | 69.2 mph |

## 4.3    WAVE CONDITIONS

Figure 4.1 shows the project site view with a breakwater partially shielding the site from waves directly from the south. As shown in Figure 4.2, the longest fetch distance to the project site is from the shores of Sausalito and approximately equals 35,400 feet (i.e., 6.7 miles). We utilized a STWAVE model to produce a complex wave climate from wind generated waves in a 100-years storm event at the project site. The STWAVE model simulates depth-induced wave refraction and shoaling, depth- and steepness-induced wave breaking, diffraction, wave growth because of wind input, and wave-wave interaction and white capping that redistribute and dissipate energy in a growing wave field. The model domain spans the shores of Sausalito to the project site to maximize the exposure length for wave build-up known as fetch distance. The model utilizes the NOAA Digital Elevation Model (DEM) with the Datum MHW.

Figure 4-3 illustrates the model domain and wave heights within the region under storm conditions. The wave analysis shows that a noticeable wave height is produced upon entering the breakwater mouth, but then subsequently diffracts into the dredged channel therefore reducing the wave energy and height. Sheltering effects from the breakwater lead to a reduction in nearly half the entering wave height to the shore. Table 3-4 summarizes the wave output from the STWAVE model at the project site. The significant wave height and period of waves propagating from extreme wind events was utilized for design check of the coastal protection along the project shoreline.

We estimated wave run-up using the van de Meer equation per the Coastal Engineering Manual by the U.S. Corps of Engineers. The equation estimates the wave run up achieved by the highest 2% of incoming waves during a storm event. Assumptions made for the project site are the roughness of the riprap, the slope of the revetment, and the waves being shore normal. The roughness coefficient is 0.55, correlating to two or more layers of rock. The slopes or the project site range from 3H:1V on the Northwestern edge, to 2H:1V on the Southeastern edge. The wave climate input used is the 100 year wind generated waves seen in Table 4-3. Table 4-4 summarizes the estimated wave run-up height. All calculations are located in the Appendix.

### Table 4-3:  STWAVE Output

| Return Period (Year) | Wave Conditions* ($H_s$ / $T_p$) S-W Waves |
|---|---|
| 100 | 3.25 ft. / 4.25 sec |

*$H_s$ – Significant Wave Height; $T_p$ – Peak Spectrum Wave Period

### Table 4-4: Wave Run-Up at the Project Site

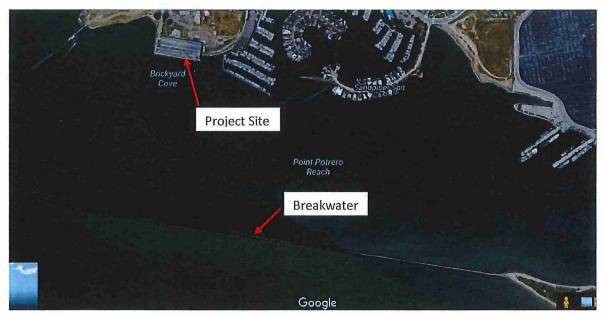| Slope | Run-up, $R_{u2\%}$ |
|-------|--------------------|
| 1H:1V | 5.4 ft |
| 2H:1V | 5.4 ft |
| 3H:1V | 4.8 ft |



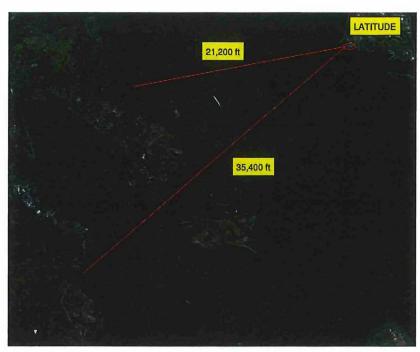Figure 4.1 Project site and breakwater
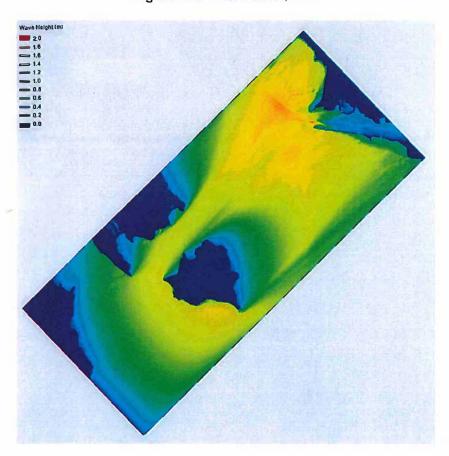
**Figure 4-2: Fetch Distances**



**Figure 4-3: STWAVE Model Output**

## 4.1 ASSESSMENT OF SHORELINE PROTECTION

Our site inspection confirmed that the rip-rap underneath the Latitude Wharf is classified as Class "Light" (200 lbs) per Caltrans' California Bank and Shore Rock Slope Protection Design Manual (RSP Design Manual). The stone underneath the wharf contains multiple rock layers and extends up to 30 feet below. The rip-rap adjacent to the wharf at the Northwest and Southeast side is in range of ¼ Ton to 1 Ton in size per RSP Design Manual. The most exposed corner at the Northwest of the wharf has a slope of about 3H:1V, while the Southeast end of the site is 2H:1V. The bathymetry measured by inspection for pile lines are given in Figure 3. The riprap was observed to extend down to depths ranging from about 3 ft. to 5 ft. below MLLW along the length of the wharf. The shoreline along the wharf extending to the southeast is shielded from the wharf piles and breakwater, which reduce incident wave heights, making these regions less susceptible to storm waves. The northwest shoreline has the highest exposure area to waves.

Using STWAVE analysis results, we performed a design check of the existing rip-rap at the site in accordance with Caltrans' RSP Manual. The engineering calculations indicated that the minimum size of the rip-rap should be Class Light even in the most exposed northwest corner of the site. Therefore, the existing rip-rap should be adequate for shoreline protection for 100-year storm. The southeast shoreline currently extends to a grade of +8.0 ft. above MLLW with RSP placed the full extent. We recommend that a splash apron using RSP Class Light be placed to extend 4 feet beyond the current RSP in the southeast region, in order to avoid possible ponding and erosion of the back soil behind and beneath the existing rip-rap.

## 8.    APPENDIX – EVALUATION OF SHORELINE PROTECTION

The appendix includes the analysis and calculations of the maximum design waves and minimum size of riprap at the project site in accordance with the design guidelines in Caltrans' California Bank and Shore Rock Slope Protection Design Manual (RSP Design Manual) and in US Army Corps of Engineers Coastal Engineering Manual (CEM).

SGH derived the design wind speed using the data taken over 45 years at the NOAA station located in the Alameda Naval Air Station. The maxima wind speed records were corrected using the guideline in CEM for the elevation of the sensor as well as the duration of the reading. We first predicted the wave period and wave heights at the project site using the approximation methodology in CEM. Figure 8-1 was used to establish the fetch distance used for the CEM method. The CEM method does not account for diffraction or refraction due to the presence of the breakwater. SGH then used a STWAVE model to obtain more realistic wave conditions at the site in the 100-year storm event. The STWAVE analysis is able to more accurately predict wave heights and period with the presence of the breakwater located near the project site. Figure 8-2 and 8-3 illustrates the model domain and project site using the Digital Elevation Model (DEM) provided by NOAA for the San Francisco Bay. The STWAVE output is shown within the model domain in Figure 8-4 and 8-5. From the output, SGH then used the RSP Design Manual to assess what class riprap was needed. The CEM was used with site investigations of slopes seen in Figure 8-6 to obtain the vertical extent needed for the riprap through run-up calculations. The following calculations represent the methods described herein and establish the basis for our assessment of the existing shoreline protection at the Latitude project site.
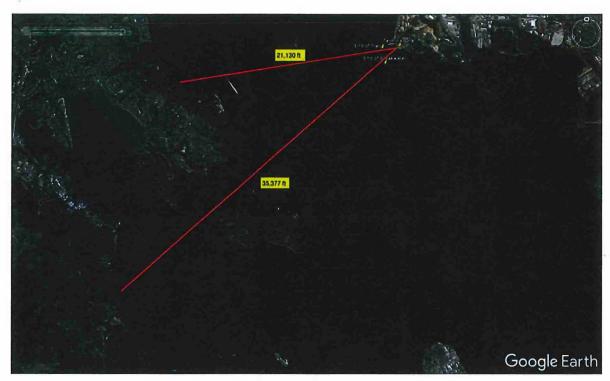
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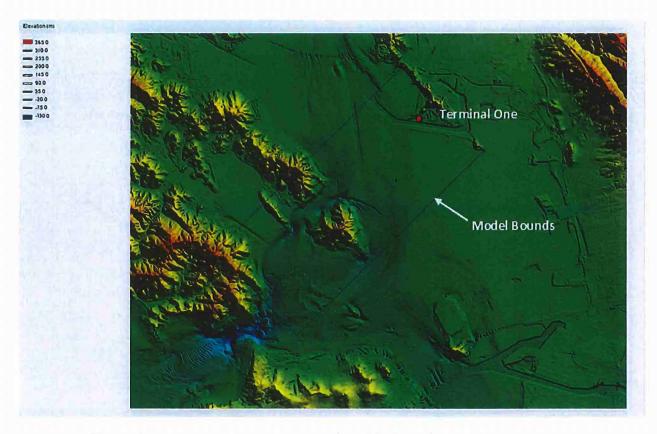**Figure 8-1: Fetch length arrays for the project site**

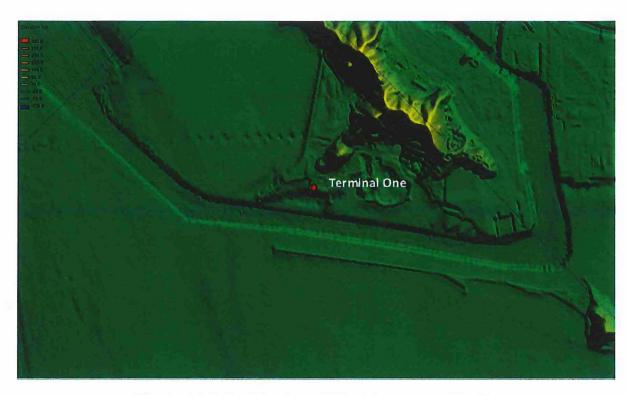Figure 8-2: Terminal One in 1/3 arc second DEM with model bounds

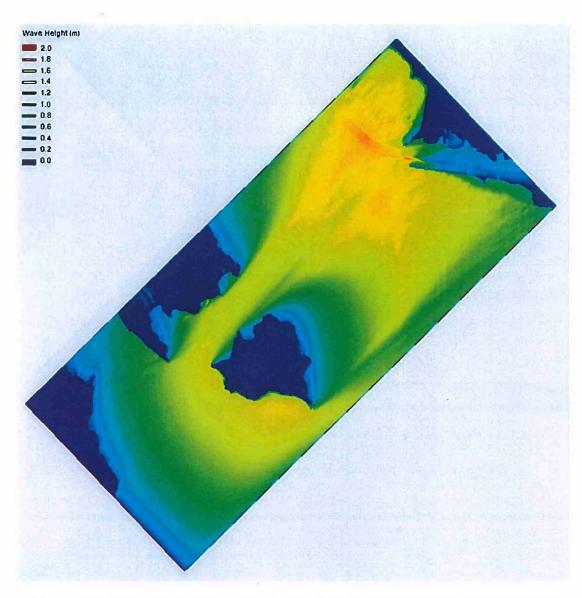Figure 8-3 Project location within 1/3 arc second DEM

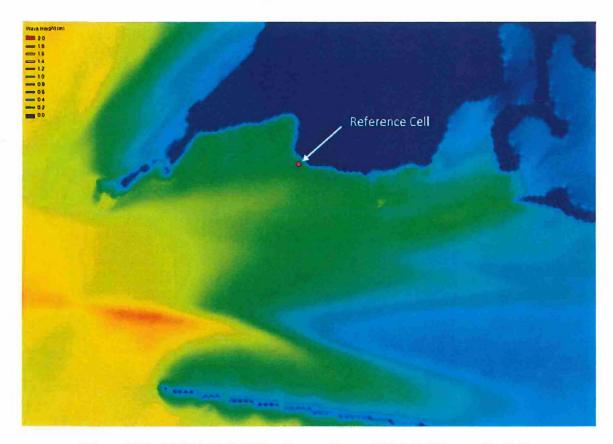**Figure 8-4: STWAVE significant wave height for 69.2 mph wind from Southwest**

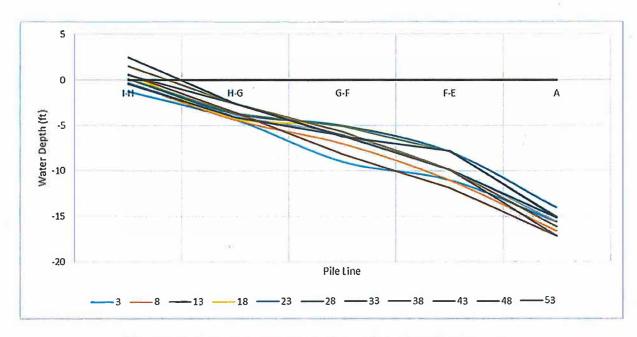Figure 8-5: STWAVE significant wave height highlighting project site

Figure 8-6: Bathymetry for pile lines along length of wharf

## Wave Hindcasting

References: *Coastal Engineering Manual, California Bank and Shore Rock Slope Protection Design, 2003*

Description: Calculation of both significant wave height and predominant waver period through CEM Methods by fetch limits

## INITIAL INFORMATION :

## INPUTS

| | | |
|---|---|---|
| Fetch Distance | $X := 35377ft = 6.7\,mile$ | |
| Unadjusted Wind Speed | $U_o := 70.52mph$ | 100 Year Storm Wind Speed |
| Measured Wind Duration | $t_o := 3sec$ | (Must be in seconds) |
| Elevation Above Water Level Where Wind was Measured | $z := 9.1m$ | Should be between 8-12 meters |
| Wind Duration Desired | $t_0 := 95.7min$ | (Must be in minutes, large enough to make fetch limited) |
| Average Water Depth | $d_o := 60ft$ | |

## CONSTANTS:

| | |
|---|---|
| Wave Height Constant #1 | $\lambda_1 := 4.13 \cdot 10^{-2}$ |
| Wave Height Constant #2 | $m_1 := \dfrac{1}{2} = 0.5$ |
| Wave Period Constant #1 | $\lambda_2 := 0.751$ |
| Wave Period Constant #2 | $m_2 := \dfrac{1}{3}$ |
| Drag Coefficient | $Cd := 0.001$ |

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SUBJECT: TERMINAL ONE WAVE
HINDCASTING BY CEM

PROJECT NO: 167573.00
DATE: 4/26/2017
BY: MLA
CHECKED BY: RI

## DURATION ADJUSTMENTS:

Correction factor for duration, t=to

$$CF := \begin{vmatrix} 1.277 + 0.296 \cdot \tanh\left(0.9 \cdot \log\left(\frac{45\sec}{t_0}\right)\right) & \text{if } t_0 \leq 3600\sec \\ 1.5334 - 0.15 \cdot \log\left(\frac{t_0}{\sec}\right) & \text{if } t_0 > 3600\sec \end{vmatrix}$$

$$CF = 1.509 \qquad \text{(CEM Figure II-2-1)}$$

Correction factor for duration, t=tD

$$CF_D := \begin{vmatrix} 1.277 + 0.296 \cdot \tanh\left(0.9 \cdot \log\left(\frac{45\sec}{t_D}\right)\right) & \text{if } t_D \leq 3600\sec \\ 1.5334 - 0.15 \cdot \log\left(\frac{t_0}{\sec}\right) & \text{if } t_D > 3600\sec \end{vmatrix}$$

$$CF_D = 1.462 \qquad \text{(CEM Figure II-2-1)}$$

Correction factor to obtain desired duration

$$CF_d := \frac{CF_D}{CF} = 0.968$$

## LEVEL ADJUSTMENTS:

Correction factor for adjusting to 10m level

$$CF_{u10} := \left(\frac{10m}{z}\right)^{\frac{1}{7}}$$

$$CF_{u10} = 1.014$$

## ADJUSTED WIND SPEED:

Adjusted Wind Speed

$$U_{10} := U_0 \cdot CF_d \cdot CF_{u10}$$

$$U_{10} = 60.2 \text{ mph}$$

## FRICTION VELOCITY:

Friction Velocity

$$u_6 := \sqrt{Cd} \cdot U_{10}$$

$$u_0 = 3.096 \text{ mph}$$

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## LIMITING WAVE PERIOD:

Limiting wave period $\qquad$ $T_{p\_limit} := 9.78 \cdot \left(\dfrac{d_o}{g}\right)^{0.5}$

$T_{p\_limit} = 13.356\,s$

## FETCH LIMITED WAVE GROWTH:

Time required for waves to cross X given wind velocity U to become fetch limited

$t_{x0} := 77.23 \cdot \dfrac{X^{0.67}}{U_{10}^{0.34} \cdot g^{0.33}}$ (CEM II-2-35)

$t_{x0} = 5.698 \times 10^3\,s$

Nondimentional Length $\qquad$ $X_{hat} := \dfrac{g \cdot X}{u_a^2}$ (CEM II-2-36)

$X_{hat} = 5.521 \times 10^4$

Nondimentional Time $\qquad$ $T_{p\_hat} := \lambda_2 \cdot X_{hat}^{m_2}$ (CEM II-2-36)

$T_{p\_hat} = 28.596$

Nondimentional Wave Height $\qquad$ $H_{mo\_hat} := \lambda_1 \cdot X_{hat}^{m_1}$ (CEM II-2-36)

$H_{mo\_hat} = 9.704$

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SUBJECT: TERMINAL ONE WAVE
HINDCASTING BY CEM

PROJECT NO: 167573.00
DATE: 4/26/2017
BY: MLA
CHECKED BY: RI

## FETCH LIMITED WAVE GROWTH CONTINUED:

Dominant Waver Period

$$T_p := \min\left(T_{p\_limit}, \frac{T_{p\_bar} \cdot u_0}{g}\right)$$

(CEM II-2-36)

Significant Wave Height

$$H_{s0} := \frac{H_{mo\_bar} \cdot u_0^2}{g}$$

(CEM II-2-36)

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## Terminal One Riprap Design Calculation

References: *Coastal Engineering Manual, California Bank and Shore Rock Slope Protection Design, 2003*

*California Department of Transportation, California Bank and Shore Rock Slope Protection Design, 2000*

Description: Riprap design due to wave runup from STWAVE analysis output

### RIPRAP DESIGN: DESIGN DESCRIPTION

The design of the shore protection revetment running along the project site is comprised of stone placed in layers to provide erosion protection of the shoreface to wave interaction. The shore protection is required to extend up above the expected water run-up height exceeded by 2% of incident waves as well as down to a depth sufficient to avoid toe scour. The weight of the riprap needs to be appropriately large enough to withstand energy transfered from design waves.

### RIPRAP DESIGN:

#### INPUTS

Significant Wave Height $\quad H_s := 3.25\text{ft}$

Maximum Wave Height $\quad H_{max} := 1.8 \cdot H_s = 5.85\text{ft}$

Significant Wave Period $\quad T_p := 4.25s$

Beach Slope $\quad \alpha_1 := \dfrac{1}{1} \qquad\qquad \alpha_2 := \dfrac{1}{2}$

$\alpha_3 := \dfrac{1}{3}$

Roughness factor for two or more layers of rocks $\quad \eta_r := .55 \qquad\qquad$ (CEM Table VI-5-3)

#### WAVE RUNUP:

Deep Water Wave Steepness $\quad s_{op} := \dfrac{2 \cdot \pi \cdot H_s}{g \cdot T_p^2} = 0.035$

Surf Similarity Parameter $\quad \xi_{op\_1} := \dfrac{\alpha_1}{\sqrt{s_{op}}} = 5.335 \qquad\qquad \xi_{op\_2} := \dfrac{\alpha_2}{\sqrt{s_{op}}} = 2.667$

$$\xi_{op\_3} := \frac{\alpha_3}{\sqrt{s_{op}}} = 1.778 \qquad \text{(CEM VI-5-2)}$$

Runup Exceeded by 2% of Waves
$$R_{u2\%\_1} := \begin{vmatrix} (1.5\,\xi_{op\_1}\,H_s\,\gamma_r) & \text{if } 0.5 < \xi_{op\_1} < 2 \\ (3.0\,H_s\,\gamma_r) & \text{if } \xi_{op\_1} > 2 \end{vmatrix}$$

$$\boxed{R_{u2\%\_1} = 5.363\,ft}$$

$$R_{u2\%\_2} := \begin{vmatrix} (1.5\,\xi_{op\_2}\,H_s\,\gamma_r) & \text{if } 0.5 < \xi_{op\_2} < 2 \\ (3.0\,H_s\,\gamma_r) & \text{if } \xi_{op\_2} > 2 \end{vmatrix}$$

$$\boxed{R_{u2\%\_2} = 5.363\,ft}$$

$$R_{u2\%\_3} := \begin{vmatrix} (1.5\,\xi_{op\_3}\,H_s\,\gamma_r) & \text{if } 0.5 < \xi_{op\_3} < 2 \\ (3.0\,H_s\,\gamma_r) & \text{if } \xi_{op\_3} > 2 \end{vmatrix}$$

$$\boxed{R_{u2\%\_3} = 4.768\,ft}$$

(CEM VI-5-7)

## REVETMENT ROCK DESIGN:

### INPUTS:

Slope of Revetment
$$\cot\alpha_3 := 2$$

$$\cot\alpha_T := 3$$

Depth of scour below mean sea level plus 1/2 maximum tidal range
$$dB := 8ft$$
3 ft. = 1/2 Tidal Range
5 ft. Depth of scour below Water Line

Specific Gravity of the Rock
$$SGR := 2.65\,\frac{gm}{cm^3}$$

Specific Gravity of seawater
$$SGW := 1.0265\,\frac{gm}{cm^3}$$

Randomly Placed Rubble Correction Constant
$$r := 70\deg = 1.222 \qquad \text{(CEM VI-5-67 Table)}$$

Stability Coefficient
$$K_D := 3.5$$

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## ROCK SLOPE PROTECTION SPECIFICATIONS:

### CAL TRANS PARAMETERS:

Minimum Rock Mass to Resist Shallow Water Wave Forces

$$W_{CT\_S\_2} := \frac{0.0003 \cdot (dB)^3 \cdot SGR}{\left[\left(\dfrac{SGR}{SGW}\right) - 1\right]^3 \sin(r - \alpha\_s)^3} \quad \text{(CalTrans Eq. 2)}$$

$$\boxed{W_{CT\_S\_2} = 222.7 \, lb}$$

$$W_{CT\_S\_3} := \frac{0.0003 \cdot (dB)^3 \cdot SGR}{\left[\left(\dfrac{SGR}{SGW}\right) - 1\right]^3 \sin(r - \alpha\_s)^3}$$

$$\boxed{W_{CT\_S\_3} = 137.4 \, lb}$$

### CONCLUSION:

Cal Trans produce a weight of rock near RSP Class "Light" rock for the shore protection being adequate for the differing slopes.

| Slope 2H:1V | Slope 3H:1V |
|---|---|
| $W_{CT\_S\_2} = 223 \, lb$ | $W_{CT\_S\_3} = 137 \, lb$ |

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